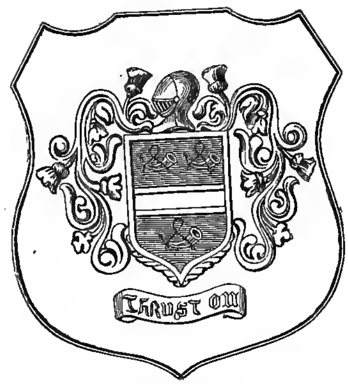
THRUST ON

City of New York.

Aqueduct Commission.

Two Reports

ON RESEARCHES CONCERNING THE DESIGN AND CONSTRUCTION
OF HIGH MASONRY DAMS, IN VIEW OF THE
PROPOSED BUILDING OF

Quaker Bridge Dam,

BY

B. S. CHURCH, *Chief Engineer*, AND A. FTELEY, *Consulting Engineer*.

1887.

REPORT

OF

B. S. CHURCH,

CHIEF ENGINEER.

NEW YORK, July 28, 1887.

To the Honorable Aqueduct Commissioners:

GENTLEMEN: I herewith respectfully submit the Report called for by your Resolutions of March 24, 1886, and June 22, 1887, on the subject of High Masonry Dams, both curved and straight in form.

The plans presented for the straight Quaker Bridge structure were made after long study of the forces and strains to be sustained, allowing for large factors of safety against overturning, crushing and sliding. The best proportions and form consistent with safety and reasonable economy were designed, with great care to meet all requirements of strength and stability.

The world's experience in the designing and construction of great dams, together with the investigations and opinions of the highest authorities, in Europe and America, were collected and considered in the plans prepared and presented to the Commission.

The strength and cost of a straight dam, resisting hydraulic pressure, ice, wind and wave action by its weight only, was compared by mathematical results with the stability and cost of a dam in the form of a horizontal arch, transmitting thrusts to the hill-sides. For the Quaker Bridge structure, the strength, economy, and safety, of the straight dam, so far exceeded what was obtainable by the curved form that I can only account for the construction on a curve of so many large modern masonry dams, from the fact that only within a few years has the question begun to receive the attention it deserves from trained and experienced hydraulic engineers and authors.

The popular belief in masonry arches to bear great weight or pressure, is justified when confined within reasonable limits. But no masonry bridge arch of over 300 feet span has been constructed.

Beyond that length of span, the crushing strain, sustained by voussoirs, exceeds safe limits on their mortar joints from the weight of the masonry alone, without the added load the bridge should carry.

It would therefore be out of the question to construct a masonry bridge of the length of Quaker Bridge Dam (1350 feet long) as it would be over four times the limit of safety in a possible span. The arch of a curved dam, however, does not carry any part of its own weight (as in the case of a bridge), therefore its strength of arch will bear only the water pressure against it from a full reservoir.

I invite your attention to the following statements on the curved form.

1st. When the section of the Quaker Bridge Dam is of a required area of cross section to resist reservoir pressure by its own weight, and is then built on a curve, the dam cannot act as an arch.

Such a section (See E F G H, Sheet B) will closely approach the shape of a right angle triangle, with its apex at the surface of the reservoir.

The water pressure against it, from a full reservoir, will be nothing at the surface, and as the pressure increases, at successive depths, a corresponding thickness of masonry in this section increases in due proportion down to its base to sustain such pressure.

In order that arch-thrust may act to transmit pressure to the hill-sides, an elastic movement of the masonry, resulting from water pressure, must, of necessity, take place to produce such thrust.

This approximate triangular section of masonry is of a form that will best resist and practically stop such elastic movement, in the plane of the cross-section, that transmits arch-thrust. It must be remembered, that the possible elastic movement of the vertical transverse cross-section is insignificant when compared with the longitudinal elastic yield through so long a dam.

In short, the stiffness of the triangular cross-section prevents the far greater longitudinal elastic movement required to permit appreciable arch-thrust to reach the hill-side.

Therefore, a long, curved dam with a gravity section will resist hydraulic pressure against it by its own weight, and not by arch action.

2nd. Let us next consider the effect of reducing the cross-section for a curved dam at Quaker Bridge, by reducing its breadth of base and thickness of masonry, to a degree that permits bending, or elastic movement, that will allow the dam to act as an arch. Assume the thickness of the dam forming the arch ring to be from 80 to 100 feet, and that it will be sufficiently elastic. We then find that, with a radius of 900 feet, we will have a pressure on voussoir joints of from 42 tons to the square foot with 80 feet breadth of section, to 39 tons with 100 feet breadth of section, on the supposition that the line of strain bisects each joint. Such pressures are, however, beyond safe limit for hydraulic masonry. Again, the elastic movement will throw the weight resultant near the

end, or outside of the base, causing a tensile strain in horizontal joints of the inside face, which should not occur, as it causes a tendency to rupture the masonry and create leakage.

It is evident, therefore, that we cannot reduce the section or thickness of so long and high a dam in order to force it to act as an arch, without causing dangerous and complicated strains.

3rd. We must next consider the effect of increasing the cross-section or thickness of the dam at Quaker Bridge, in view of broadening the voussoir joints for reducing the thrust pressure of a curved dam of 900 feet radius, to 16 tons to the square foot, the limit of pressure in the straight gravity plans, which have been submitted to you. We find that it produces a section (A B C D Sheet B) that is nearly double that required for a straight gravity dam, (E F G H) having proportionately less tendency to act as an arch, than in the first mentioned case; while, at the same time, it adds 100 per cent. to the cost.

It, therefore, becomes evident that it is impossible to construct a curved dam at Quaker Bridge which will act as an arch to any apperciable degree, and consequently there is no valid reason for risking the complicating and serious rupturing strains, which a curved dam will produce; or, for the increased expense it would entail.

In order to verify the above statements and figures, I have worked out a formula, and used it for the calculations on curved dams herein given, verifying them by the formula of moments, where that formula became applicable. (See Sheet A. Fig. 1).

In a curved dam having a radius (R) sustaining a weight of water pressure (W) and total pressure (p) on a voussoir joint of any span or cord (C), then we have the formula ($p=\frac{R.W.}{C}$).

The demonstration of the above formula is as follows: (Fig. 1).

Let A E and B E be drawn perpendicular to the radii A D, and B D, and tangent to the curve A W B. Take E g equal to the weight (W) of water pressure on a belt of the dam one foot in breadth, and the length of the cord.

From (g) draw g h and g i parallel to E B and E A. Then will E h g i become a parallelogram of forces, and E h equals one component, normal to the voussoir joints at A, and equals the total pressure (p) on that joint,

The triangle E h g is similar to the triangle A D B, their sides being perpendicular to each other, therefore A B : E g : : A D : E h, or C : W : : R : p, and $p=\frac{WR}{C}$.

The radius, weight and cord, or span, being known, the pressure p is found by

this formula, which being divided by the area of the joint in square feet gives the average pressure per square foot.

This formula proves the fact that, with a given radius, the total pressure on each of the voussoir joints is equal and remains the same to all lengths of span, from the length of a voussoir, to the total length of the dam, and that the pressure on voussoir joints increases as the length of radius increases. If we assume a value for (p), we can find the value of R by the above proportion, which becomes $R=\frac{C.\ p.}{W}$.

Now, if we assume a rectangular section, equal to the area of the section presented for the straight Quaker Bridge Dam, it will be 101 feet in breadth, limiting the pressure to 16 tons to the square foot, then (p) or the total pressure on a voussoir joint, ten feet long on the extrados cord, will be 1616 tons at a depth of 107 feet below the surface and (W) will be 51 tons for the same span of ten feet on a belt one foot broad.

Then, by the formula, $R=\frac{C.\ p.}{W}$ equals 316.8 feet as the length of the radius required to maintain the conditions, This radius will not be long enough to span the Valley at the Quaker Bridge site, even with a half circle.

In other words, with the same area of section in the better shape to act as an arch (because it admits of greater elastic movement), and with the same limit of pressures that exists in the plans before you for the gravity Quaker Bridge Dam, the radius must be so short that the dam cannot span the Valley. To further illustrate the above statements and call your attention to another important matter bearing on the subject of the Quaker Dam, I will suppose that the Quaker Bridge Dam be built on a curve having a radius of 900 feet; that instead of being a dam, it is a wall 250 feet high, making a complete circuit forming a well, resisting water pressure on its entire exterior circumference, 260 feet in depth, as in Figure 2.

The voussoirs all being of equal size it is evident that the hydraulic pressure (W) normal to the outer cord of each of the voussoirs, will be the same at any fixed depth below water surface, and the total pressure on all the voussoir joints will be equal.

The thrusts of all the voussoirs will balance each other and produce an equilibrium of voussoir pressures in any horizontal plane, at any depth, all around the circle, because the elastic movements to produce these pressures is perfectly balanced. The pressure of (a), (Fig. 2) towards (b) equals (b) towards (a), and the pressure of (e) towards (f) equals the pressure of (f) towards (e). etc. Now a hill-side abutment may be substituted for voussoirs (a) or (e).

Therefore with the same radius and section of dam, increasing the span, does not increase the total pressure on voussoir joints as above stated.

In the complete circle, however, perfectly balanced elastic movement takes place to produce uniform pressure on the several voussoir joints.

But when the thrust of a curved dam is sustained by unyielding hill-sides, the hill-sides do not respond to the elastic movement (as in the case of voussoirs in the complete circle), consequently the line of transmitted resultant pressure through all the voussoirs cannot pass (as in the complete circle) uniformly through the same part of the voussoir joints, but will take a direction approaching to the line, d, d, d, in Fig. 3, because of the tendency of the arch to flatten, and take the form of the dotted lines in Fig. 3; therefore, with a fixed radius, although the total pressure on the voussoir joints does not increase with the span, the resultant line of transmission of thrust does increase crushing strains.

It causes the line d, d, d, to fall near the ends of the joints tending to greatly increase the strains, so unevenly distributed on the different joints, and also tends to buckle and crack the arch, as well as to crush.

No experimental data or formulæ exist to enable us to measure these arched dam-thrust strains with any degree of accuracy.

It is probable that the line of strain near the centre and ends of the dam will fall outside of the middle third of the joints, more than doubling the crushing strain of 42 tons, given in the second case mentioned above, or over 82 tons to the square foot.

Therefore, the building of any arched dam must be regarded as experimental in character; it is far safer and less expensive to avoid such complications and build straight dams to resist hydraulic pressure by their weight only.

The most reliable formulæ for giving the maximum pressure on the outer and inner toes of a gravity dam are not perfect, because no adequate experimental data exist to give the elasticity of large masses of masonry and enable that factor to be included in the formula, but we know that by neglecting such factor the calculated strains exceed what will actually take place, and that we will, therefore, be within safer limits than the formula indicates. Whereas, in the calculations for curved dams, the reverse is true.

Many of the high dams we have obtained records of, are built on a curve, but all except one act as gravity dams, and their arched form must weaken their stability.

The only existing curved dam which can and does act as an arch, is the Zola Dam, 205 feet long, 126 feet high, 41 feet broad at the base.

I believe that sooner or later it will fail, becoming ruptured and weakened by

the action of changing pressures from the drawing out and refilling of its reservoir storage.

To sustain the above statements, I will refer to the following authors: Pelletreau advises that the arch form should not be used when the dam is more than 131 feet in height and length.

Delocre believes that the structure would cease to act as an arch when the thickness of this arch would reach one-third of the radius, and calculates the limit to be reached at a depth of 14.7 feet below the flow line and confines it to very short spans.

Krantz gives the limit of the span at 131.2 feet with a radius of not more than 65.6 feet.

Rankine suggests that in very short spans, a slight curve be introduced to prevent the bending out of the top of the dam by water-pressure against it; but states that he has no confidence in calculations for stability for arch-thrust in long dams, and intimates that the arch form is of doubtful utility even in short dams. He insists that the section should be such as to resist overturning by its own weight alone.

Mr. E. S. Chesbrough was sent to Europe by the Department of Public Works to examine and report upon the principal dams existing there. From his examinations and studies during his trip, he pronounced against the curved form; but his subsequent illness and death prevented the completion and presentation of his report. I can, however, testify to his stating that opinion to me while I was associated with him as Consulting Engineer in the Department of Public Works.

McMasters holds that, having established a profile that will resist the pressure of water by its own weight, there is no valid reason for constructing it in the form of an arch, and thereby increasing its length and the cost of its construction.

The standard authorities, both in Europe and in this country, agree that in long and high dams the sectional profile should be capable of resisting water pressures against it, by its own weight; and that the curve increases the cost without adding to its strength.

In fact, these general considerations militating against the arch have so satisfied them that they have not considered it necessary to go into more elaborate demonstrations.

Inasmuch, however, as the curved form has been pressed by several speakers during the public hearings before this Commission, it becomes desirable that a careful statement be given you of the inapplicability of a curved form for long and high dams.

The horizontal, or bed joints in the stone masonry constructions are always more perfectly filled and are stronger than the vertical joints. The weight of each

stone maintains its bed and, as the load of superincumbent masonry increases upon it, these joints become more firm and compact.

With the vertical joints this is not the case. In the straight dams, therefore, the most complete and strongest bed joints bear all the strain from the reservoir waterpressure against the dam, and the vertical joints of lesser strength are relieved from strain, while the reverse is true of the curved form.

The plans for the Quaker Bridge Dam, submitted to this Commission, have received, as its magnitude and importance deserve, a longer and far more thorough consideration and study than has been given to any existing dam, and I, therefore, urge their acceptance with confidence.

Figure (4). Sheet A will illustrate some static principles relating to gravity dams, wherein A B D is the form of a theoretical dam, calculated to exactly sustain the overturning pressure of water against it.

There being no factor of safety against overturning, the resistance of the dam and the pressure of the water are in exact equilibrium.

With these conditions, the resultant of pressure strikes at the end of a horizontal joint, and the dam takes the shape of a triangle, however high the dam may be. This, under the supposition that no arbitrary limit of pressures be fixed for the outer and inner toes, and that the tendency to crush is ignored.

Let us now introduce a factor of safety of (2), against overturning, by adding to the breadth of the dam to increase the length of the joints.

This correspondingly increases the bulk and weight of the masonry to a degree that will require exactly twice the water pressure (to produce overturning) that a full reservoir can ever exert.

With these conditions, and ignoring excessive pressures, the dam again takes the form of a triangle, as shown on Figure (4) by the lines A B, A C, C B.

But if (with masonry weighing 156.25 pounds to the cubic foot) we limit the pressures at the toes of the dam to 16 tons per square foot, that pressure will obtain at the line a a, 205 feet from the surface of high water.

Below that point, the limiting of pressure to 16 tons per square foot will cause the straight lines of the triangular form to bend into the curves a b E and a c F maintaining uniform pressure below that point, lengthening the joints beyond the ratio of the triangle which produces curves.

We will next consider the reasons for departing from the last described theoretical section A F E, to that of the section under consideration, shown by the outer dark lines of Fig. (4).

In the theoretical section the resultant forces strike exactly at the end of the middle third. Any additional strain of ice, or wave and wind action would, therefore, produce tensile strains on the inner face as above stated.

To prevent such strains, the section must be broadened lengthwise with the joints, to distribute the increased weight.

Again, as the maximum pressure will be on the reservoir side, or inner toe, during construction, before sufficient age has given the mortar time to thoroughly harden, we must make a second increase of section to obtain smaller pressures than will exist at the outer toe with a full reservoir.

The maximum strain with a full reservoir will not come upon the outer toe for months after the structure is completed and the mortar is well hardened.

The weight of the gravel and water resting on both toes of the dam up to the river bed at g h i and g′, h′, i′ (not considered in the theoretical section) calls for a third increase.

To maintain the limitations of pressures fixed upon, the back pressure below the river-bed against the outside of the dam (not considered in the theoretical section) demands a fourth variation, which slightly diminishes the section.

The fifth variation, increasing the area of the section, is required to thicken and strengthen the outer toe.

We now come to the last cause of change from the theoretical section, at the top of the dam. This sixth increase of section and weight on the diagram (shown at A k t) is required to resist ice, wind and wave action.

The addition of this weight enable us to encroach upon the theoretical section at t,–t′,–t″– without reducing the several factors of safety below the limit fixed; while it adds to the required breadth of section throughout the lower portion of the dam.

The strain sheet, and its tables, exhibit how evenly all the factors of safety are maintained in the final section, under consideration, providing for all the above-mentioned disturbing conditions.

As no demonstration could be found of Sazilly's trapezoid of pressure for great dams, I have developed one and obtained a formulated and graphic method of calculating the pressures on the outer and inner toes of joints of high masonry dams, and used them to re-calculate the pressures on the strain sheet obtained by the formula of moments, as a check on those results.

By this method, as in all others, we must neglect the factor of elasticity in masonry, which affects the results but slightly, and introduces a factor of safety. The demonstration is as follows: (See Fig, 5).

When the resultant of horizontal and vertical pressures cuts the joint or base A, B, in the middle, all the vertical components must equal each other, and each will equal the total weight of masonry (W) divided by the length (l) measured by the lines A, D, g h and B S. Then the trapezoid of pressures becomes a rectangle and the resultant will cut the base at the same point g, where a vertical line passing through the centre of gravity of the rectangle meets the base.

When the resultant cuts the base, A B, at the end of its middle third (k), then the aggregation of all the vertical components will form a right-angle triangle I A B, measuring the total weight of masonry, and consequently equal in area to the rectangle A B S D. The resultant will also cut its base at the same point k, at which a vertical line passing through its centre of gravity will cut the base. This triangle must have a height B I, equal to twice g h, and equal to $\frac{2\,W}{l}$ and will equal the pressure (p) at B, the outer toe of the dam, while the pressure on the inner toe A, becomes zero.

If the resultant should cut the base outside of the middle third, the pressure at A would become a minus quantity producing undesirable tensile strains. This illustrates the reason for confining the intersection of the resultant within the middle third of the base. When the resultant cuts the base within and at any point other than at the middle, or at the end of the middle third, a trapezoid will be formed which must be equal in area to the rectangle or the triangle. A vertical line dropped through the centre of gravity of this trapezoid and also the resultant must cut the base at the same point.

In the rectangle of pressures when the resultant cuts the centre of the base both p and $p' = \frac{W}{l}$.

In the triangle, when the resultant cuts the end of the middle third $p = \frac{2\,W}{l}$ and $p' = o$.

To find the pressures p and p′ when the resultant cuts the base at some other point (m); produce g h the vertical component of the rectangle at the middle of the base, upward through Z and perpendicular to D S. At the point k, where the resultant of the triangular area of pressures cuts the end of the middle third of the base, project a line perpendicular to A I until it meets g h produced at Z. From m draw the straight line m Z, and through the intersecting point h (common to the upper sides of all possible trapezoids of pressure) draw the line o t, perpendicular to m Z. Then will o B equal p, and t A equal p′ the pressures sought.

The sides of the triangles k Z g, are perpendicular to the sides of I S h. These triangles are therefore similar. For the same reason, the triangles m Z g and o h, S, are similar.

Representing the lines Z g by (a), k g by (b), S h by (c), I S by (d), m g by (e), and o S by (n).

Then

$$a : b :: c : d$$

and

$$a : e :: c : n$$

Therefore by alternation

$$b : d :: e : n$$

and

$$n = \frac{de}{b}$$

The triangles o S h, and t D h being similar and equal; and the line B, S being equal to S I or d; when the dam resists water-pressure against it.

$$(1) \qquad p = d + \frac{de}{b}$$

and

$$(2) \qquad p' = d - \frac{de}{b}$$

With no water-pressure against the dam

$$(3) \qquad p = d - \frac{de}{b}$$

and

$$(4) \qquad p' = d + \frac{de}{b}$$

In the above formulae $b = \frac{l}{6}$, $d = \frac{W}{l}$ $e =$ the distance of the centre of l from the point where the resultant cuts it.

GRAPHIC METHOD.

Having fixed the point Z, by the method above described at the intersection of g Z and k S, from the point m where the resultant cuts any part of the base l connect the points m and Z by a straight line. Through the point h draw a straight line perpendicular to m Z, cutting B S and A D (or these lines produced) at o and t, then will

$$p = B\,o$$
$$p' = A\,t.$$

By making one-sixth of the base measure $\frac{W}{l}$ in tons to the square foot (u being the end of the middle third) then the triangles Z g K and h S I, also Z g m and h S o, are not only similar but equal, and $p = u\,m$ and p' K m, measured directly from the strain sheet, when a scale one-sixth of the base measures and equals in tons $\frac{W}{l}$.

The same data being required for both the formula and graphic method, makes it convenient to check one by the other.

As Consulting Engineer, A. Fteley could best give time to arrange, collate, and present to you the information we have obtained respecting existing dams, called for in your resolution. I have requested him to embody those matters in his report, together with any further detailed technical presentations that might suggest themselves to him.

The details for the tightest mode of bonding, by breaking joints vertically as well as horizontally, the systems of masonry in construction, and a method of securing tightness on the interior face, I have prepared for contract and working drawings to be made, when the Commission has settled on the main features of the plans now under consideration.

TABLE.

SECTION OF QUAKER BRIDGE DAM, ACTING AS AN ARCH 900 FEET RADIUS, CALCULATED BY FORMULA $p = \frac{W R}{C}$

Depth below Surface.	Span or Cord. C	Radius. R	Pressure on 10 foot Span. w	Total pressure on a voussoir joint 10 feet long on extrados. p	Depth of voussoir joints.	Pressure per sq. ft. on voussoir joints.
Feet.	Feet.	Feet.	Tons.	Tons.	Feet.	Tons.
0	10	900	0.000	000,00		0.00
10	10	900	3.125	281.25	19.7	14.73
20	"	"	6.250	562.50	21.0	26.74
30	"	"	9.375	843.75	23.1	36.52
40	"	"	12.500	1,125.00	27.5	40.99
50	"	"	15.625	1,406.25	31.6	44.18
60	"	"	18.750	1,687.50	36.0	46.87
70	10	900	21.875	1,968.75	44.0	44.51
80	"	"	25.000	2,250.00	51.5	43.68
90	"	"	28.125	2,531.25	59.5	42.55
100	"	"	31.250	2,812.50	67.1	41.91
110	"	"	34.375	3,093.75	77.6	39.91
120	"	"	37.500	3,375.00	88.4	38.17
130	10	900	40.625	3,656.25	98.5	37.11
140	"	"	43.750	3,937.50	109.4	35.99
150	"	"	46.875	4,218.75	120.5	35.01
160	"	"	50.000	4,500.00	131.3	34.28
170	10	900	51.125	4,781.25	141.5	32.60
180	10	900	51.256	4,613.04	152.7	30.21
190	"	"	51.256	4,613.04	164.0	28.13
200	"	"	51.256	4,613.04	174.0	26.51
210	"	"	51.256	4,613.04	185.0	24.93
220	10	900	51.256	4,613.04	195.5	23.59
230	"	"	51.256	4,613.04	204.5	22.55
240	"	"	51.256	4,613.04	212.0	21.75
250	"	"	51.256	4,613.04	216.0	21.35
260	"	"	51.256	4,613.04	216.0	21.35

The results of the above table are graphically exhibited in sheets (B) and (C).

Very respectfully,

B. S. CHURCH,

Chief Engineer.

REPORT

OF

A. FTELEY,

Consulting Engineer.

NEW YORK, July 25th, 1887.

B. S. CHURCH,
Chief Engineer, Aqueduct Commissioners.

SIR:

The following is a brief synopsis of the research on the question of high masonry dams, which you requested me to make for the ultimate purpose of determining the forms, dimensions, materials and mode of construction of the proposed Quaker Bridge Dam.

It is of such importance for the welfare of the future water supply of New York that this subject should be exhaustively studied, and the amount of money involved is so large, that the necessity of acquiring a full knowledge of the practice followed by other engineers in similar instances, became at once evident; their methods were carefully considered and weighed, and the best practice was followed, as much as practicable. Owing, however, to the unprecedented magnitude of the proposed dam, which gives rise to some important problems heretofore unsolved, it became necessary, in some instances, to leave the beaten path and to resort to a basis of operations different from that established by others,

It should be said here that the subject, when submitted to us, was not entirely new, and that we had the advantage of having before us the result of the joint-work of Messrs. Isaac Newton, Chief Engineer, and E. S. Chesbrough and J. W. Adams, Consulting Engineers, of the Department of Public Works, formulated in the official plan of Quaker Bridge Dam, submitted by the Department of Public Works to the Aqueduct Commissioners.

It is not within the scope of this report to consider the advisability of the construction of Quaker Bridge Dam as an essential part of the new system of water supply for New York; the opinion of high engineering authorities should place beyond a reasonable doubt the necessity and feasibility of the structure, but additional confidence in its ultimate success must be obviously increased by a consideration of the high masonry dams now in successful operation, although of less magnitude. The most important ones have been represented in section and

in plan on sheets Nos. 1 and 2, which accompany this report. Table I. gives statistical data in regard to the greatest number of them, and does not require any explanation; in many cases, data which appeared of interest for the purpose of comparison, were obtained by calculation from plans and other sources of information at hand, but they are in every case designated by proper references.

A glance at sheet No. 1 shows a great variety in the shape of the transverse sections of the various dams. The older Spanish Dams, Puentes, Alicante, Val del Inferno, etc., are conspicuous by their heavy sections; it is plain that their constructors did not design them in accordance with the best principles, and that they have, by accumulating masonry, uselessly increased the pressures on the lower part of the dams. From these increased pressures, however, which in the Almanza Dam attain fourteen tons per square foot, we may derive the proof that the low limits of pressures, recommended by the more modern authorities, can be safely exceeded, as the old Spanish Dams (with the exception of the Puentes which failed from a different cause) have been in long and successful operation.

The modern French type is represented on sheet No. 1 by the following dams: Furens, Villar, Ban, Habra, Komotau, Ternay, Hamiz, Gran Cheurfas, etc; it is almost needless to add that this form, first indicated by Mr. Sazilly, is due to a better understanding of the magnitude and direction of the forces acting on a dam at various stages of the water in the adjacent basin, and that it is from the construction of these dams, and from the indications of their designers, that the most useful information can be procured.

Other modern dams, the Gileppe, the Vyrnwy, the Zola, present special features which shall be considered in time.

Before entering into the consideration of these various structures, however, it is proper, in order to comply with the request of the Aqueduct Commissioners, that I should refer, as succinctly as possible, to the authorities which have been consulted on the subject, and to the views advanced by them.

We know little or nothing of the methods used by the engineers who built the old Spanish Dams to determine their proportions. The design of the more modern dams, built later in France, for the water supply of canals, "show (says Mr. Krantz) a clearer conception of the forces to be overcome, and greater care in regard to economy. In a word, they show a real advance in the art of building. Still they are not irreproachable. It is obvious that they cannot be taken for models."

Mr. Sazilly was the first * to consider the subject in all its bearings. "With rare insight,** he was able to discern the nature, amount, and direction of the various

* Annales des Ponts & Chaussées, 1853.

** Krantz, p. 21.

forces which act on reservoir walls at the different periods of filling the basins. From them he deduces rational forms to be given to constructions of this sort."

His deductions led him to a profile of a stepped section, which he advocates for several theoretical and constructive reasons; he recommends a limit of pressure on the masonry of about six tons per square foot, and shows his profiles to a height of 160 feet. His method of calculation is intricate and requires, as usual in such cases, a series of approximations; he assumes that the pressures are distributed in accordance with formula (1 and 2) page 25, which seems to have been universally adopted to express the distribution of pressures on any horizontal joint of a dam, and which will be mentioned again in the course of these remarks.

Mr. Delocre* developed a very complicated method for determining the profile of high masonry dams, which he applied to the design of Furens Dam, admits a limit of pressure of about six tons per square foot, and does not confine the point of application of the resultants to the middle third of the joint; he assumes in his calculations for the masonry a weight of 125 lbs. per cubic foot.

Mr. Graeff,† who had charge of the construction of the Furens dam, must be mentioned as the author of a description of that structure.

Mt. Krantz ‡ gives an interesting and comprehensive review of the subject of high masonry dams, and suggests a full series of profiles for dams of various heights, on which the ultimate pressures are about 6.1 tons per square foot. He does not develop the methods which led him to the determination of his profiles.

His exposition of the comparative sections of high masonry dams is very complete.

§ Professor Rankine, taking occasion of his being called upon to report on the construction of a proposed dam in India, gives his views on the subject of high masonry dams. He calls attention for the first time to the necessity of using a lower limit of pressure on the front than on the back face, for the reason that the pressure upon a curved face of masonry, at any point, is tangent to the curve at that point.

He also argues on the advisability of keeping the resultants within the middle third of any joint, in order to avoid tension, which, he claims, would otherwise exist at some points of the faces.

Finding also that, for dams not exceeding 150 feet in height, logarithmic curves give a close approximation to a correct profile of the faces and furnish a rapid method for determining the outline of a Dam section; he recommends their use.

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* Annales des Pontes & Chaussées, Oct., 1866.

† Annales des Ponts & Chaussées, Sept. 1866.

‡ Study of Reservoir Walls, by J. B. Krantz, translated by F. A. Mahan, 1883.

§ Miscellaneous Scientific Papers, 1881.

* Mr. Bouvier, in an article on the Ternay Dam, claims that fomula (1 and 2) p. 25, used to compute the distribution of pressure on the base of dams, should be so modified as to make the resulting pressure depend upon the oblique resultant, in the case of a full reservoir, thus increasing the pressures as otherwise calculated.

This method accomplishes the suggestion of Prof. Rankine to adopt a lower limit of pressure for the front face. Mr. Bouvier departs from the practice of his predecessors, and admits, on the strength of the crushing resistance of rubble masonry and of hydraulic lime mortar, as shown by Mr. Vicat, that the pressure on masonry can be safely increased to 14 1-3 tons per square foot.

† Mr. Pochet gives a description of the Habra Dam in Algeria, refers to the poor quality of the materials used and, after giving a list of pressures which obtain in various cases, adds that the limit of pressure of six tons per square foot, generally admitted by engineers, must not be taken as absolute, and that, in his belief, experience will lead to a much bolder practice within the limits of safety.

‡ Mr. Pelletreau, after a purely theoretical discussion upon the designing of high masonry Dams, submits a mathematical method no simpler than that advanced by Mr. Delocre.

§ Mr. de Beauve has devised a graphic method for determining the profile of high dams ; his method considers only the limiting condition of pressure, and it requires considerable modification to introduce the condition of confining the point of application of the resultants to the middle third of the base. The determination of a profile by this method is very laborious, but exact.

Mr. Guillemain, Professor of the School of Ponts and Chaussées, Paris, in an unpublished lecture delivered at that school, reviews the whole subject of designing high masonry dams ; he considers that the points of application of the resultants should be confined within the centre third of the base, and he takes the same views of the computation of pressures distributed on the base of a dam, as formulated by Mr. Bouvier, and suggested by Professor Rankine.

‖ Mr. McMaster gives some mathematical equations for the determination of a profile ; he also devotes considerable space to certain practical considerations in regard to the building of masonry dams.

** Mr. Aymard, having access to the early public records concerning the older Spanish dams, has made use of that information, in connection with his examina-

* Annales des Ponts & Chaussées, Aug., 1875.

† Annales des Ponts & Chaussées, April, 1875.

‡ Annales des Ponts & Chaussées, Oct., '76, Aug., '77, Mar., '79.

§ Manuel de l'Ingenieur des Ponts & Chaussées, translated by E. Sherman Gould, C. E., 1881.

‖ " High Masonry Dams "; Van Nostrand's Science Series, No. 22.

** Irrigation du Midi de l'Espagne.

tion of the structures, to give a complete account of them; interesting facts connected with the construction and use of the dams are found in the work.

* Mr. Crugnola, in his exhaustive treatise on retaining walls, devotes a considerable space to masonry dams; he shows the sections of a number of dams, and gives a graphic method for determining the lines of resistance in any assumed profile.

The author recommends typical sections for various heights up to 164 feet.

The following documents, bearing on the subject, have also been consulted, viz.:

The manuscript report of Mr. Bideau to the Minister of Public Works of Belgium, on the plan of the Gileppe Dam. A memoir on the same structure by Messrs. Detienne and Leclerk.

Miscellaneous notes on the same dam [Proc. Inst. C. E., *Engineering News*, etc.]

An article on high masonry dams by Major Tulloch [Professional Papers on Indian Engineering].

Proc. Inst. C. E. Vol, 56, p. 101, on the description of a dam built of concrete in Australia for the Geelong Water Supply.

Proc. Inst. C. E. Vol. 71, p. 379, giving a short description of the Villar Dam.

Scientific American, May 28th, 1887. A historical and descriptive review of earth and masonry dams, with plans, by David Gravell.

A report by Mr. Geo. F. Deacon, C. E., as to the Vyrnwy Dam for the water supply of Liverpool, with full account of the methods used for the determination of the profile and for the construction.

A paper by W. B. Coventry, Proc. Inst., C. E., Vol, 85, Paper No, 2110.

Annales des Ponts & Chaussées, June, 1872. A detailed and illustrated description by Mr. Pamairesse of the Barrage de Cheliff.

Annales des Ponts & Chaussées, Vol. XI., 1886. A theoretical discussion by Mr. Hetier upon the designing of the profile of masonry dams.

Giornale del Genio Civile, 1885; also, Bacini di alimentazione per cura di G. Torricelli, C. E.

Annales des Ponts & Chaussées, June, 1872. A detailed description of the Masonry Dam on the "Canal du Verdon," in France, by Mr. de Tournadre, with note describing the Barrage de Zola, 124 feet high.

Torricelli.—Relazione presentate al Ministero di Agricoltura.

Lagrené.—Cours de Navigation—II.

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* Muri di Sostegno. G. Crugnola.

Die Thalsperre der Gileppe bei Verviers (F. Kuhn, Dresden).

Der Civilingenieur for 1879.

In addition to these special references, it is obvious that other classical and well known authorities have been consulted.

When considering the exceptional magnitude of the proposed Quaker Bridge Dam, it becomes at once apparent that it is necessary to depart in its design, to a considerable extent, from the practice hitherto followed. The accumulation of ponderous solid masonry with the great and ill defined strains to which it must be submitted is at best, although the only advisable one under the circumstances, a rude and expensive way of solving the problem, and some attempt was made to see whether, in this case, a structural building could not be erected, with its various parts so designed as to resist well known strains, and to render it comparatively easy to locate and remedy defects, if any were found, after construction.

This study led to the form of an inclined plane, built of masonry of sufficient quality and thickness to resist the percolation of water, and supported on groin arches and stone pillars ; the junction of the structure with the sides and bottom being effected by means of heavy masonry walls penetrating the rock to suitable depths.

Whatever may be the objections to such a structure, it may be seen that all its parts would be subject to well known strains, and would be left open to inspection ; this research, however, was not extended beyond its preliminary stages, because it soon became evident that the masonry pillars and other parts supporting the structure, should be made of such cut stone masonry as could support the heavy weight reposing on them, and that the total cost of the dam would be unreasonbly high.

An iron structure built on the same principle would be feasible and economical, but would not present such guarantees of permanence as must obtain for a dam which is to be the main reliance of the future water supply of New York.

An earth dam, with a core of masonry, was also considered. In such structures, the thrusts exerted by the earth against the centre wall, during construction and at the different stages of the water surface, when filling or emptying the adjacent basin, are unknown or, at the best, very uncertain ; if the wall became fractured under their action, or allowed the water, in places, to pass through ; the leaks, under such a considerable and unusual pressure, would endanger the stability of the earth embankment. Moreover, the authorities agree that this kind of structure is not safe to adopt beyond a certain limit of height, and it was thought unwise to attempt an experiment which, owing to the importance of the interests involved, was not justified by the saving in expense which might have been effected by its adoption.

In this case, where the welfare of New York is concerned, and where the consequences of a possible disaster with 30,000,000,000 gallons of water behind the dam, may well be imagined, we must deal with certainties; we have before us many successful instances of masonry dams, although of less magnitude, and the proposed structure can be so proportioned and constructed in masonry as to stand effectually the great pressure to which it is to be subjected.

After this preliminary consideration of the subject, it was consequently decided, in conformity with the opinion expressed by the Engineers of the Department of Public Works, to build the dam of solid masonry with such differences from previous engineering practice as would be necessitated by the special features of the undertaking.

In designing the profile of the dam, three well known main conditions are to be observed.

1st. The point of application of the resultant of the forces acting on the dam, whether or not it be under water pressure, must not be beyond the centre third of the base.

2d. The pressures on the masonry at either face, and at any point, must not exceed a certain safe limit.

3d. The sections of the dam, at any point, must be such that no sliding can take place.

The means used by the various authorities, analytically or graphically, for the determination of profiles fulfilling wholly or partly these conditions are somewhat different, but the greater number having based the computation of the distribution of strains on the lower joint of the dam or on any horizontal joint, on the method reproduced by Mr. De Beauve* in his "Manuel de l' Ingénieur," it is proper that attention should be called to it.

The pressure at each end of the base or joint "is given by one or the other of the two formulæ

$$(1)\ p = 2\left(2 - \frac{3u}{l}\right)\frac{W}{l}$$

$$(2)\ p = \frac{2}{3}\,\frac{W}{u}$$

according as u is greater or less than $\frac{l}{3}$."

* Manuel de l' Ingenieur des Ponts & Chaussées. Translated by E. Sherman Gould, C. E., 1881.

l = length of base or joint.

p = pressure per unit of surface at one end of base or joint.

u = distance from the point of application of the resultant to same end of base or joint.

W = total pressure on base or joint.

This method is based on the hypothesis that the masonry forming the dam is absolutely rigid and leads to the conclusion that the maximum pressures, on any base, take place at one end of it when the reservoir is full and at the other end when the reservoir is empty. But the structure is not rigid, and there is obviously some uncertainty in regard to the distribution of the strains through the mass of the masonry. This view of the subject is illustrated in the following extract of a report of Mr. J. B. Francis to the Chief Engineer of the Department of Public Works.

"The main part of the dam, I am informed, is intended to be of rubble masonry laid in Portland cement.

"Although the work may be done in the best manner possible, it will yield a little to the pressure, and the amount of the compression in different parts of the dam will be unequal and nearly proportional to the pressure and depth of the mass of masonry at its different parts. The amount of the compression at different points may be too small to be observed, but I am of the opinion that it will be sufficient to modify to a material extent the result of calculations based on the hypothesis of an incompressible mass, and the effect, I think, must be to cause a distribution of pressure very different from that indicated by the calculations, and to relieve to a great extent, if not entirely, the outer toe of a high dam designed strictly according to the De Beauve method, from the pressure due to the weight of the high part, combined with the horizontal pressure of the water against the up-stream face of the dam. The entire weight must, of course, be supported on the base, but I think a much larger proportion of it will be upon its central parts, than is indicated by the calculation, and that the pressure at the toe, instead of being a maximum, will be a minimum."

As an additional illustration of the uncertainty of the distribution of pressures on the base of dams, the following extract from Rankine's "Miscellaneous Scientific Papers," may be of interest: "The assumption on which this rule is based, of an uniform rate of variation of that component of the pressure which is normal to the pressed surface, is known to be sensibly correct in the case of beams, and is probably very near the truth in walls of uniform, or nearly uniform, thickness. Whether, or to what extent, it deviates from exactness in walls of various thicknesses is uncertain in the present state of our experimental knowledge."

Whatever may be the distribution of the pressure on the base of the dam, it must be admitted that the strains are distributed otherwise than indicated by the formulæ; but as we have no other means of determining these strains with accuracy, and as, on the other hand, the formulæ indicate resulting pressures larger than they are in reality, the profiles based on that method show an excess of strength which gives a further guarantee of safety.

The distribution of strains on the base and horizontal joints of the dam has consequently been computed, in the numerous cases considered, by formula [1], with the understanding that it indicates, at the ends of the joints, pressures evidently higher than the actual ones, thus adding to the computation a factor of safety of important, although uncertain, value.

The sections of dams built or proposed by various authorities have been determined by methods involving the consideration of the horizontal and vertical forces acting on the dam, and of the magnitude, direction, and application of their resultant force. Those methods, with the exception of some analytical ones mentioned before, and which are too complicated for practical use, proceed by successive approximations of the profiles, and require considerable time. As it was desirable, in this research, to go thoroughly over the ground and review a number of different profiles before coming to final conclusions, a more expeditious method was sought and used for a large portion of the preliminary studies. This method, although proceeding by successive approximations, was found of great service for simplifying the computations. It is based upon the principle of moments.

For convenience and accuracy, the profile is divided into horizontal layers of a given thickness. The calculation is begun at the top of the dam, each layer being figured separately. A layer being given, and the profile of the dam above it being known, certain dimensions are assumed for the lower joint of the given layer; and the position of the two resultants (in case the reservoir is full or empty) are determined by taking moments about assumed axes. If the first profile does not give results in accordance with the assumed limiting conditions, another is attempted, and a third attempt is very rarely needed.

To simplify the computations the weight of the cubic foot of masonry is taken as the unit of weight throughout the calculations; consequently, when considering a vertical section of the dam one foot thick, the area of the section in square feet will give the weight in cubic feet of masonry; hence laborious reductions are avoided.

A practical example of the method will illustrate it.

The upper part of a dam m, n, a, b, Fig. 5, Sheet No. 5, being already known,

it is desired to determine an additional section 20 feet high. The depth of water above the base of the new part is 150 feet.

The weight of masonry is assumed at 156,25 lbs., per cubic foot. (This weight is within less than a pound that of a block of masonry built for experimental purposes with stone quarried in the neighborhood of Quaker Bridge Dam, and has the advantage of being a single multiple (2.5) of the weight of a cubic foot of water.) The limiting pressure for the back face is supposed to be 20,625 lbs. per square foot (10 kilos per square centimetre) = 132 units, expressed in cubic feet of masonry; the limiting pressure on the front face being 105 cubic feet of masonry.

We want to find first the point of application of the resultant when the reservoir is empty. The resultant W for the part above joint a b is known in magnitude and position = 5,689.5 cubic feet masonry. The axis of moments is 192.5 feet from point b.

From the previous form of the profile, let us assume that on the front face we will add d k = 21 feet to the base a, b, and on the back face c, l, = 4 feet.

Divide the trapezoid a, b, c, d, into one rectangle a, k, b, l, and two triangles b, c, l and a, d, k.

Taking the moments about the assumed axis

	Weights in cubic feet of masonry.	Lever Arms.	Moments.
Resultant W	5,689.5	224.1	1,275,388
Rectangle 93 × 20	1,860	239.	444,540
Triangle 21 × 10	210	292.5	61,425
Triangle 4 × 10	40	191.2	7,648
Totals	7,799.5		1,789,001

$\frac{1,789.001}{7,799.5}$ = distance of the resultant from the axis of moments = 229.4 ft.

Distance of the point of application of the resistance from point c = 229.4—[192.5—4] = 40.9 ft.

In the case when the reservoir is full, the pressure of water will throw the resultant toward the front edge.

Calling h the depth of water above c d, the moment of the water pressure around point d. $= \frac{h^2}{2} \times \frac{h}{3} \times \frac{1}{2.5} = 225.000$, $\frac{1}{2.5}$ being the ratio of the weight of water to weight of masonry.

Then taking moments about the resultant found in the first case, $\frac{225{,}000}{7{,}799.5} =$ 28.9, which is the distance between the points of application of the two resultants.

Applying now formula (1) it will be found that the pressures at d and c are 101.6 and 126.7 cubic feet of masonry, respectively, inside of the assumed limits of pressure. If greater accuracy is needed a new trial may be had with slightly lower values for d, k and l, c.

The results thus obtained have been frequently checked by graphic methods.

In the various computations of profiles, which will be here mentioned, the vertical pressure of water on the back face of the dam is omitted; its omission has only a slight influence on the results, which it affects on the side of safety, and simplifies the calculations.

It may be noticed that in the course of these remarks, the height of Quaker Bridge Dam has always been computed, from the bed rock, many feet below the river bed, to the top of the structure. It is obvious that the position of its lower part in a deep bed of gravel, and the counter-pressure due to the earth and to the water on the front face, modify to a certain extent the conditions of stability and the intensity of the strains, but no additional strength is given to the dam by the fact of its being imbedded in a great depth of earth, and with due regard to the modifications which may be introduced on account of the presence of a counter-pressure, the dam must be treated as one structure of a uniform character throughout its whole extent.

At the beginning of this research, a trial profile attempted under conditions of assumed pressures within the limits used by the best authorities, showed conclusively that the height of Quaker Bridge Dam would compel us to depart from the practice of others, as, in order to remain within the prescribed limits, it would be necessary to spread the lower parts in an impracticable manner and to incline the slopes to an extent incompatible with strength. Fig. 1, Sheet No. 5, illustrates this fact.

The same result would follow the use of the Rankine method and of the types advocated by Messrs. Krantz, Crugnola and others (see sheet No. 3).

It was then thought that it might be advisable to change the quality of the masonry at the points a, b, Fig. 2, Sheet No. 5, where the application of the pressures recommended became impracticable, and to adopt beyond these points a cut stone masonry capable of supporting greater strains. Further consideration of the case, however, developed the fact that this modification would be very costly owing to the size of the dam; it is also preferable not to destroy, if possible, the homogeneity of the mass of masonry.

The only alternative left was to assume higher limits of pressure. Is it safe

to do so? In answer to that question, we may refer again to the opinion of Messrs. Bouvier & Pochet (see page 6), who admit that bolder practice can be resorted to within safety. It must be said also that, in fixing their low limits of pressure, the French engineers have kept in view the fact that the dams may be put under full pressure when their comparatively slow setting mortars have not gained their full strength, while in our case the cements used, with the possible exception of the uppermost part of the dam, will have time to harden satisfactorily before the whole force of the water pressure is brought to act on the masonry. Moreover, it is well known that there are in existence many structures, the foundations of which have to support weights superior to any that may be sustained by the lower parts of Quaker Bridge Dam.

A computation made of the maximum pressure at the outer toe of some existing dams shows that they attain in several cases from eleven to fourteen tons per square foot.

It is well known that the ultimate crushing strength of the materials to be used in Quaker Bridge Dam is much above the pressure which will obtain in that structure, and it must not be forgotten that the limits of strength deduced from experiments are obtained with samples of a small bulk, presenting much less resistance to crushing than large masses of masonry laterally supported.

Adding to these considerations the fact already mentioned, that the pressure on the faces of the dam are undoubtedly below the figures calculated by the general method followed by us, it may be asserted that the pressures calculated for the lower part of the proposed structure can be safely adopted.

The question may then be fairly asked whether these high limits of pressure, inasmuch as they are considered safe, should not be adopted for the upper part of the profile of the dam, as well as for the lower part. They could, in our opinion, be adopted, but an insignificant saving only would thus be effected and some objectionable features would be produced. The upper part of the dam, which has to resist the additional strains due to the shock of waves and ice and to the action of frost, possibly at a time when the mortar is not fully set; and which has to carry a roadway, must be designed much wider than it would be if it was subjected only to the combined pressure of masonry and of quiescent water; such being the case, the thicknesses of masonry corresponding to the higher pressure assumed for the base, could not be adopted near the top without distorting the profile; if adopted lower down, their effect would be to cause the lower outer slope of the profile to make with the horizontal a sharper angle. Considerable importance is attached to this point; the downward regular transmission of strains due to superincumbent weight in a pyramidal mass of masonry can be relied on within certain limits only, and it is obvious that it would not take place if the slopes were

too much inclined on the horizon ; the safe limit of acceptable slope is uncertain and depends on the character of the materials and of the workmanship which enter into the structure.

It may be added also that it is better, *where practicable*, to follow the practice which has been proved by experience to be successful in dams already built, and to depart from it in the parts only to which it is not applicable, and in which further safety is obtained by the greater extent of the bulk of masonry (with its consequent additional resistance to crushing due to increased lateral support, and its additional strength due to the increased surfaces of cement joints).

The lowest layers of the masonry, at the time that the dam will be put under pressure, will also have the advantage of the harder set of the mortar due to the time necessary for the building operations.

The acceptance of this basis of operation disposes of one of the essential conditions previously mentioned for the proper determination of the profile, *i. e.*, that the point of application of the resultants on the base of the dam must be within the centre third of the said base, as, owing to the spreading of the latter to correspond to the assumed limiting pressures, this condition obtains in all cases. Let it be added now that the third condition, *i. e.*, the resistance to sliding, need not require any subsequent attention ; in all cases the horizontal water pressure tending to make the dam slide on its joints is much inferior to the weight of the masonry, and, when taking in consideration the adhesive power of the cement mortar and the roughness of the surfaces in contact, it becomes evident that no sliding motion can take place.

The weight of the masonry is an important factor in the establishment of a profile, and it can be seen that the weights [actual or assumed] entering into the calculations of the various authorities vary considerably.

Several weights of the cubic feet of masonry have been used in this research ; 156.25 lbs. has been used for the proposed profile of Quaker Bridge Dam, having been determined, as mentioned before, by actual experiment.

The change of weight, however, has much less influence on the profile than might be supposed, as is seen from an inspection of fig. 3, Sheet No. 5, in which the comparative profiles have been designed under the following conditions :

	Weight of one cubic foot of Masonry.	Limit of Pressure per square centimeter.
Profile a.	125 lbs.	8 and 10 kilos.
" b.	145.8 "	7.6 " 9.8 "
" c.	156.25 "	8 " 10 "

The weight of the masonry being for the present an uncertain factor, it was useful to see to what extent the condition of stability would be changed, in case a dam, calculated for a given weight of masonry, were afterwards constructed with materials of a greater or lesser weight. A profile was consequently designed using 156.25 lbs., as the assumed weight of the cubic foot of masonry, and a subsequent computation of the pressures made on the supposition that the actual weight of the masonry was 125 lbs.

The same operation was repeated, 125 lbs. being the calculated weight, and 156.25 lbs. the actual weight.

In the first case the pressures were found as follows for the front face :

Depths from top.	Calculated Pressures. Pounds per square foot.	Actual Pressures. Pounds per square foot.
30 feet.	11.981	10.750
50	13.260	13.000
70	14.882	15.750
90	15.678	16.625
110	16.116	16.500
130	16.224	16.125
150	15.850	15.675
171	16.146	15.994
288	38.376	35.400

In the second case, the pressures were found as follows, also for the front face :

Depths from top.	Calculated Pressures. Pounds per square foot.	Actual Pressures. Pounds per square foot.
30 feet.	8.888	9.984
50	10.625	10.000
70	11.338	11.177
90	12.250	12.383
110	13.875	13.803
130	15.827	15.912
150	16.250	16.068
171	16.500	16.378
288	33.460	35.724

These results, obtained with weights so far apart, show that a small error in the expected weight of the masonry forming the dam would not seriously affect the reliability of the profile.

While on the subject of the weight of the masonry, it is proper to consider whether the fact that a portion of the dam, from the river bed to the bed rock below, is to be submerged in water, is to affect the weight of the structure.

It would obviously be so, to a certain extent, if the masonry was porous enough to admit water under ordinary hydrostatic conditions, but the dam, at the lower part, is to be so thick and the masonry is to be of such quality that it can be reasonably supposed that the water penetrating it will be mostly, if not wholly, in a capillary condition, and that the whole dam may be considered, for purpose of calculation, as an impervious monolithic mass rigidly and imperviously connected with the rock bottom, and, consequently, not losing weight by the fact of its being in water. It must be added, however, that even if admitting, as it has been by others, that the water would exert an upward pressure on the dam, thus diminishing its actual weight on the bottom, the result would not be such as to disturb, to any objectionable extent, the conditions of stability otherwise established.

This subject leads naturally to the consideration of the advisability of building drains in the mass of the dam, as recommended by others, in order to relieve it from the pressure of water penetrating the masonry. In accordance with the preceding remarks, and with some authorities, their presence is not thought necessary, and it is obvious that, unless their construction is superintended with the greatest care, they may become injurious by introducing water into the masonry under conditions which would then become unfavorable. At any rate, we know by the experience of existing dams that they have been built and work successfully without drains, and we fail to find any sufficient reason to discard that practice.

In the preceding remarks, the conditions of stability referred to have been considered in connection with the base of the dam or with its horizontal joints only, but the tendency of the parts of the dam to rotate about the foot of diagonal joints assumed at various angles, through the mass of masonry, has also been examined, and, in all cases, the moment of the masonry around the lower point of the given diagonal joint has been found more than double of the moment of water pressure around the same point.

The conclusions herein formulated in regard to the pressures that it is thought advisable to allow on the base of the profile, and in regard to the slope of the front face, indicate obviously that, in order to determine the profile of the lower part of Quaker Bridge Dam, the approximate length of the base and the maximum pressures at the toes, must be arbitrarily fixed within the limits which have been shown here to be safe.

The result of successive approximations demonstrates that by adopting for maximum calculated pressure at the end of the base fifteen tons per square foot (a value which, in view of the opinions previously expressed in regard to the distri-

bution of pressures on the base, is considered safe), the length of the base of the theoretical profile will be about 230 feet, a length which allows of a front slope likely favorable to the transmission of the strains acting upon the structure from its various points to its base.

In consequence of these views and of the considerations already developed in regard to the limiting pressures adopted for the upper part of the profile, and in view of the comparatively large proportions of the extreme top, it follows that the calculated pressures at the ends of the successive horizontal joints will show progressive values from the top to the base.

The few sloping lines forming the outline of the profile, which were adopted to facilitate construction, follow closely the many sided polygonal figure which would be indicated by the successive calculations of a number of horizontal slices of the section.

In the course of these studies Mr. Ed. Wegmann, Jr., Special Assistant Engineer, who was intrusted with the mathematical computations connected with the subject, devised a simple and exact analytical method for determining directly the profile of a masonry Dam.

That method is based primarily on the fundamental formulæ for distribution of pressure already mentioned [p. 25], and on the limiting conditions, viz.: that the resultant when the reservoir is full or empty, shall lie within the centre third of the base of the profile; and that at no point of the faces shall the elementary pressures exceed certain fixed limits.

For the purpose of calculation, the section of the Dam is supposed to be divided in horizontal slices of a given height, to which the method is applied in succession.

The vertical component of the water pressure on the slopes of the up-stream face is neglected.

Reference to the units of weight, etc., employed in the calculations for the Quaker Bridge Dam will be made hereafter.

The symbols employed are as follows:

$W' =$ Weight of masonry above the layer under consideration.

$W = W'$ plus weight of layer under consideration.

$m =$ Length of upper joint, already determined.

$l =$ " " lower " to be "

$2h =$ Depth of layer.

$d =$ Distance of point of application of W' from back end of joint m. [Reservoir empty.]

$u =$ Distance of point of application of W from front end of joint l. [Reservoir full.]

u' = Distance of point of application of W from back end of joint l. [Reservoir empty.]

p = Elemementary pressure at front end of joint l. [Reservoir full.]

p' = Elementary pressure at back end of joint l. [Reservoir empty.]

M = moment of water pressure about the horizontal plane through joint l.

a = Vertical distance of flow line below top of profile.

δ = Ratio of unit weight of masonry to unit weight of water.

The method as applied to a general profile consists of two initial equations.

First initial equation (*a*).—The first initial equation [a] applies to that portion of the profile in which the back face is vertical and covers the case in which the limiting pressure obtains on the front face. It gives the length l of the joint.

Equation [a.] $$l^2 + \frac{2\,W'l}{p} = \frac{6\,d\,W' + 2\,h\,m^2 + 6\,M}{p}.$$

If the resultant [reservoir full] falls at the extremity of the middle third of the joint, equation [a] becomes equation [b].

Equation [b.] $$l^2 + l\left[\frac{2\,W'}{h} + m\right] = \frac{3\,d\,W}{h} + m^2 + \frac{3\,M}{h}.$$

Equation [b] again transformed, becomes equation [c_1], which gives the depth at which the front face ceases to be vertical [assuming a certain top width for the profile]; *i. e.*, the section being a rectangle of given base, it gives the depth at which the resultant [Reservoir full] will intersect the extremity of the centre third of the joint.

Equation [c_1.] $$h^3 - \frac{\delta\,h\,l^2}{4} = \frac{\delta\,a\,l^2}{8}.$$

Equation [c_1] assumes that the top of the profile is *above* the level of the flow-line; if, on the contrary, the top of the profile coincides with the flow-line, Equation [c_1] becomes Equation [c^2].

Equation [c^2.] $$h^2 = \frac{\delta\,l^2}{4}.$$

Second initial Equation (*d*).—The second initial equation [d] applies to that portion of the profile in which both faces have a batter, and covers the case in which the limiting pressure exists on both faces of the profile.

Equation [d.] $$l^2[p + p' - 2\,h] - 2l[W' + h\,m] = 6\,M.$$

If the resultant, when the reservoir is empty, falls at the extremity of the middle third of the joint, then equation [d] becomes equation [e].

Equation [e.] $$l^2 = \frac{6\,M}{p}.$$

If both resultants [Reservoir full and empty] fall at the extremities of the middle third of the joint, then equation [e] becomes equation [f].

Equation [f.] $$l^2 + l\left[\frac{W'}{h} + m\right] = \frac{3\,M}{h}.$$

It is obvious that since equations [d], [e] and [f] apply to that portion of the profile in which both faces have a batter, the length of the joint [l] derived from them does not fix the batters of the layer in hand. This renders necessary the supplementary equation [g] by which the offset (x) on the back face is determined.

Equation [g.] $$x\,[3\,W' + 2\,h\,m + h\,l] = W'\,[2\,l - 3d] + hm\,[l - m] + l^2\left[h - \frac{p'}{2}\right].$$

If the resultant [Reservoir empty] falls at the extremity of the middle third of the joint, equation [g] may be simplified to the form of equation [h].

Equation [h.] $$x\,[3\,W' + 2\,h\,m + h\,l] = W'\,[l - 3d] - hm^2.$$

The above equations apply to a general profile. In the Quaker Bridge Dam, for reasons which will be explained hereafter, very long, straight slopes have been adopted for the lower portion of the profile; and the calculations for a theoretical profile were complicated by the fact that some 87 feet of the structure is below the river-bed, and consequently subjected to additional strains due to the weight of the overlying gravel, and by a counter-water pressure. This special case has been submitted to analysis and the following equations have been deduced.

The depth of the gravel overlying the base is represented by 2 g.

Total depth of layer = 2h.

Ratio $\frac{g}{h} = \theta$

Ratio of unit weight of gravel to unit weight of masonry = β.

The limiting conditions being that certain pressures shall obtain on the up-stream and down-stream faces at bed rock, equation [k] gives the length of the base at bed rock.

Equation [k.] $$l^2\Big[[p' + p] - 2\,h\,[1 + \theta^2\beta]\Big] + 2\,l\,[h\,m\,[\theta^2\beta - 1] - W'] = 6\,M.$$

Equation [l] gives the offset [x] on the back face.

Equation [l.] $$x\Big[h\,l\,[1 + 2\,\theta^3\beta - 3\,\theta^2\beta] + h\,m\,[2 - 2\,\theta^3\beta] + 3\,W'\Big] =$$
$$h\,l^2\,[1 - \theta^2\beta + \theta^3\beta] - h\,m^2\,[1 - \theta^3\beta] + h\,l\,m\,[1 - 2\,\theta^3\,\beta + \theta^2\beta] -$$
$$3\,W'd + 2\,W'l - \frac{p'\,l^2}{2}.$$

With the exception of formula $[c_1]$, a simple equation of the third degree, it may be noticed that all the others are of the second degree only. In regard to the last two, which may appear too long for practical purposes, it can be seen that they are of easy solution, and that they cover the special case when the lowest calculated section is partly built in gravel overlaying the bed rock, with a counter pressure of water.

The following table shows the application of the above formulæ to the theoretical profile for Quaker Bridge Dam:

Elevations of Joint.	Limiting Conditions.		Equations Employed.	Values Found.	
	Front Face.	Back Face.		For l	For x.
171.3	$u=\frac{l}{3}$	....	$[c_1]$	$l=20$ $(2h=34.7)$	
156.	$u=\frac{l}{3}$	....	[b]	26.2	
136.	$u=\frac{l}{3}$	....	[b]	37.4	
116.	$u=\frac{l}{3}$	$u'=\frac{l}{3}$	[f] & [h]	53.4	2.3
96.	p = 8.2 tons... per sq. ft.	$u'=\frac{l}{3}$	[e] & [h]	71.2	2.0
—52.	p = 15 tons... per sq. ft.	p = 15 tons per sq. ft.	[k] & [l]	231.8	25.4

It is by this method that the theoretical profile of Quaker Bridge Dam, shown (Fig. 4) Sheet No. 5, has been computed; the results have been afterwards confirmed by means of graphic and other methods. The elements of the calculation are as follows:

The limit of pressure assumed at the down-stream face of the profile is, for a depth of water of 110 feet (elevation +96) and less, 8.2 tons of 2,000 lbs. per square foot.

At the back or up-stream face, the condition that the resultant should fall on the end of the centre third of the base led to a pressure, at the end of the horizontal joint, at elevation +96, of 8.7 tons per square foot.

From 110 feet depth (elevation +96) to the base of the dam (elevation —52) the assumed pressures increase steadily until they attain the maximum value of 15 tons (30,000 lbs,) already mentioned.

For the sake of simplification, the vertical weight of the water on the inclined portion of the up-stream face is neglected, this slight error being on the side of safety.

The vertical weight of the saturated gravel resting upon the slopes of the dam below the river bed is taken into account.

The gravel on the down-stream side of the dam being saturated with water, the resulting counter-pressure of water enters into the computation.

The weight of a cubic foot of water is taken at 62.5 lbs.

The weight of a cubic foot of masonry, for reasons already given, is taken at 156.25 lbs.

The weight of a cubic foot of saturated gravel is taken at 145.875 lbs., equivalent to 0.94 of the weight of masonry [the dry gravel being assumed at 125 lbs., with voids equal to one-third of the bulk filled with water].

The unit of weight used throughout is the weight of one cubic foot of masonry. It has been already stated that this choice of unit simplifies the calculations greatly.

The effect of wind pressure on the dam has been neglected. If it blows down the valley when the Reservoir is empty, its tendency is to increase the stability; if it blows in the other direction the result is the same with a full reservoir, and with an empty or partially empty basin the margin of safety is more than sufficient to counteract the comparatively insignificant effect of the wind pressure.

The result of the calculation is shown on Fig. 4 (Sheet No. 5), which gives the theoretical profile of the dam, and on Table II., which explains itself:

The theoretical profile of the dam being determined, certain modifications were advisable to meet some practical requirements of adaptability and of construction.

The width of the top had to be adapted to the requirements of a roadway. A slight addition to the thickness of masonry down to Elevation 170 (*i. e.*, the part covered by the fluctuations of the water surface in the Reservoir) presents the double advantage of increasing the strength of the structure at a point where it may have to sustain shocks from the action of the waves and ice, and of adding to the symmetry of the profile.

It is also advisable for the purpose of facilitating construction to so regulate the various inclinations of the faces as to give them a simple ratio easy to follow during the building operations.

Last, as no dependence can be placed on the strength of the sharp triangle of masonry forming the extreme toe of the down-stream face, it is advisable to dispense with it, and to round it off in the form of steps, as shown on Sheet No. 6.

This disposition presents the additional advantage of lessening the width of the expensive excavation to be made in the bed of the river.

These modifications having caused some changes in the distribution of the materials, obviously rendered necessary a new computation of the distribution of pressures throughout the mass of masonry.

To this end the proposed profile was divided into horizontal sections or slices 11 feet high. The centres of gravity were found by the usual system of moments for each joint; the overturning moments, etc., were ascertained, and the pressures computed as before in accordance with the formula (1), p. 25, the various units and elements of calculation being the same as those used for the computation of the theoretical profile.

Table III. gives a synopsis of the results obtained, and shows that:

The total width of the base is 216 feet.

The maximum pressure at toe of up-stream face is 15.4 tons per square foot [13.8 tons of 2240 lbs.].

The maximum pressure at toe of down-stream face is 16.6 tons per square foot [14.8 tons of 2240 lbs.].

Average pressure on base, per square foot, 10.49 tons.

In accordance with the views already discussed in this report, these results are acceptable, and the profile, shown on Sheet No. 6, is recommended for the acceptance of the Aqueduct Commissioners.

Before closing these remarks on the profile of high masonry dams it is proper that we should refer to several modern structures which, owing to their magnitude and to the known skill of the engineers who designed them, were duly considered by us, namely: the Gilleppe Dam in Belgium and the Vyrnwy Dam near Liverpool, England, (See Sheets No. 1 and 2).

The evidently excessive section of the Gileppe Dam was due, as its designer admits, to local reasons of expediency independent from technical considerations.

The Vyrnwy Dam is intended to be used as a weir, and its shape is such as to lead the water properly in its fall and to support a heavy apron on the down-stream side.

The special conditions under which these dams were designed do not obtain in the case of Quaker Bridge Dam, and there was no valid reason to follow their peculiar profile.

A dam has been recently begun near San Mateo, for the Spring Valley Water Company of San Francisco (California), which will be almost as high as the Furens Dam, its height being 170 feet (See Sheets Nos. 1 and 2). It is to be 20 feet wide at top with straight batters. It is to be built of concrete blocks formed in

place with a mortar of Portland cement. The description of this structure, now at hand, does not mention whether the engineer has more confidence in concrete than in stone masonry, or whether his choice was determined by economical or other reasons foreign to the constructive value of these materials.*

After determining the profile of Quaker Bridge Dam, the next question in importance is to decide on the plan of the dam, whether it should be straight or curved.

Referring to the list of dams now in operation, it will be seen that both plans have been followed with successful results. If we consult the various authorities on the subject, we find the almost unanimous opinion that if a curve be adopted for the plan of a dam, it is advisable, notwithstanding, to so proportion the profile of the section as to make it able to stand, by its own gravity, the pressure of the water; the curved form, in other words, is considered as a sort of additional co-efficient of safety.

We fail, however, to see how a mass of masonry, proportioned to stand water pressure by its own gravity, can at the same time act as an arch; if the profile is sufficient, the mass will not yield under pressure, and if it does not yield, no-pressure can be transmitted to the sides through the curve; the two conditions are incompatible.

If, on the contrary, the section of masonry is not sufficient to support the water pressure, then the mass will yield to a certain extent under the strain, and some pressure will be transmitted to the abutments of the curve, *in the upper portion of the structure;* we say, in the *upper portion* because it is evident that, if there is, through the adhesive power of the mortar, rigid connection between the dam and the foundation rock, the adjacent parts of the masonry cannot, at the same time, act as an arch.

Among the curved dams recorded herein we find that all, with one exception, have sections sufficient to resist the water pressures brought to bear upon them, and that they do not act as arches. The exception is the Zola Dam [in France]; see Sheets Nos. 1 and 2 and Table I.

The thickness of the masonry is so small that the resultant falls outside of the front toe and that (unless the excellence of its mortar, and, generally, of its masonry be exceptionally great), its stability is due to the fact that it is built in the form of an arch, the water pressure being transmitted to the abutments. Moreover, the form of its section shows that the intention of its designer was that it should act so.

As, however, the rigid connection of the dam with the rock of the foundation

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* See N. Y. Daily *Tribune*, October 2d, 1887.

precludes, at that point, the idea of an elastic motion in the masonry and of a consequent lateral transmission of the strains to the abutments, it is impossible to state at what height above the foundation the dam begins to act as an arch.

Owing to the small width of the valley (a few feet at the bottom and about 200 feet at the top) the strains thus produced are evidently such as can be sustained by the masonry, but it is probable that the failure of one part of this dam would cause its total ruin, while, with a self-supporting section, a dam may fail at one point, leaving the balance whole. Such was the case at the dam of Gran Cheurfas (Algeria).

If we leave this special instance to return to general cases, we must call attention to the important fact that *the admissibility of the arch form in the plan of a dam must depend primarily on the intensity of the lateral or horizontal strains that would be transmitted through the structure.*

In the case of curved dams thrown across *narrow valleys*, it may be found that the lateral strains (admitting that any should be developed), would be such as the masonry could safely support, and, if it is expected that the upper part of the structure might be put under pressure when the mortar, not being wholly hardened, is in a comparatively yielding state, we conceive that a certain amount of motion of the masonry, however small, may be expected, causing the portion affected to act as an arch with the tendency of closing the joints under pressure.

This advantage can hardly be claimed, however, for structures which are not made of voussoirs, but composed of an irregular mass of stones and mortar, which cannot be relied on for arch work, unless the whole is sufficiently hardened to present an homogeneous and monolithic character.

When, on the contrary, the width of the valley is such that the strains transmitted by an arch would become excessive, that form must be avoided.

Let us look at the case in hand.

Great care has been taken in designing the profile that no strains superior to fifteen tons should be produced by the combined action of the water pressure, and of gravity; other strains will be developed by variations of temperature; additional local strains will occur, owing to unavoidable irregularity in the quality of the workmanship. Would it be prudent to complicate matters by knowingly submitting the masonry to additional horizontal strains which would be very high, attaining a maximum of over forty tons per square foot; as far as we can calculate such values, for "in the present state of science, the calculations of stability, treating the dam as a horizontal arch, are so uncertain as to be of doubtful utility." The best information we have in regard to the stability of stone arches comes from experimental data, and we have none bearing on arches of such a great span.

In accordance with these views it is concluded:

That a dam of such a length as Quaker Bridge Dam, if acting as an arch, would be subjected to excessive strains.

That, with the profile recommended, it can successfully resist in all its parts, the pressures applied to it, without any additional support.

That, if built on a curve, it would not act as an arch, and would be more expensive.

That it is advisable that the dam should be built in a straight line.

Let us consider, now, other features of importance, second only to the profile and plan of the dam, i. e., the height of the crest of the dam above the water line and the capacity of the weir over which the surplus water is to flow.

Existing dams furnish good data in regard to the height to be given to the structure, and in accordance with precedents and with the recommendations of reliable authorities, the top of the parapet has been placed at an elevation of 13 feet above the ordinary water mark, equivalent to 7 feet above the extreme high water mark, the surface of the roadway being 3 feet below the top of the parapet. This height is amply sufficient to cope successfully with the highest waves which may be expected from the unusual depth and considerable extent of the reservoir.

As to the capacity of the overflow, it is necessary to depart from precedents on account of the extent of the water-shed and the comparatively heavy rainfalls that occasionally occur in the Croton Basin.

Judging from the possibilities of rain or thaw in this and neighboring water-sheds, the flowing capacity of the overflow should not be less than equivalent to the flow, in twenty-four hours, of a volume of water represented by a uniform thickness of six inches over the whole water-shed.

It is true that no such flow is on record, and the actual flow may never, it is hoped, come to that figure; but a combination of adverse circumstances, such as an exceptionally heavy rainfall, occurring at a time when the reservoir is already full, and when the ground is covered with snow, can bring about such a condition of things, and it is wise to be prepared for it. It can be so much more readily done, that an increase in the length of the overflow can be obtained at a comparatively small cost.

The writer, having had occasion to design the overflow of several dams on an equivalent basis, may be allowed to state that, on the occurrence of a freshet which produced a flow somewhat less than one-half of the quantity just mentioned, he could not but feel, in accordance with the sentiment of the people living lower down in the valley, that the channels of discharge were none too large.

The length of overflow corresponding to these conditions is about 1,300 feet.

It may be mentioned here, that the gate house proposed to be built in connection with the dam, is to contain, besides the gates leading to the two aqueducts, some ample channels for the emptying of the reservoirs. These discharging channels, although placed a distance above the river bottom, are so arranged as to allow of the drawing of the bottom water, when necessary to do so.

In regard to the kind of masonry to be used, the majority of existing structures appear to be built of rubble; they generally give satisfaction, and there is no reason to depart from that practice. Concrete cannot be depended upon as being impervious enough to prevent percolation of water under great pressures and cut stone masonry, in connection with the great bulk of the dam, is obviously excluded on account of its cost. To all probabilities, stone suitable for the work can be found in abundance in the neighborhood, gneiss or granite being predominant; but in any case, the stone used must be one that will not disintegrate under the influence of time or of the weather. A proper selection of stone is of such moment, and the distance from the work to the quarry or quarries is so important an element of cost (considering especially the quantities to be transported) that the territory in the vicinity of the dam should be examined by excavations, and, if necessary, by boring, in order to determine, before letting the work, the localities from which the stone is likely to come. Such an expenditure would be judicious, and would remove, to a great extent, the uncertainty under which the contractors would labor in making their bids, and the necessity under which they would find themselves to provide in their bids a large margin to meet possible contingencies of excessive cost in transportation.

The whole masonry of the dam, from bed rock to the top, should be homogeneous with the exception of the parts which, from requirements of ornamentation and maintenance, may be made of finer masonry.

Mr. Krantz says that "the joints should be irregular on the faces and in all the sections; the courses should be thoroughly interlocked, or better still, there should be no courses, and by means of good work this result must be obtained, so that the whole body of the wall shall be a real monolith."

Mr. E. S. Chesbrough, in a report on the subject to the Department of Public Works, states that:

"The superstructure should be composed throughout of uncoursed, broken range rubble masonry, built of quarry stone of irregular sizes, laid with full beds and joints. The use of cut stone should be confined to such appropriate ornament as may be desired, none being used in the body of the work, not only on account of expense, but also to avoid the introduction of two different classes of masonry.

"While, as above stated, coursing should be avoided, it would be proper to

keep the work roughly levelled up over its entire length, say every five feet in height, so as to secure an even settlement as the structure goes up."

To these remarks, which appear to resume the practice generally followed, it may be added that, for the purpose of keeping the faces in good order, and to preserve them from the action of the weather, it is advisable to make the joint closer on the faces or even to use range rubble with a close joint near the surface only, thus lessening the future work of keeping the outer joints well pointed, without disturbing the uniform settlement of the whole mass. In some of the old Spanish dams the faces are even lined with cut stones which are not reported to have caused any trouble.

As it is of vital importance that all spaces between the stones be filled completely with mortar, they must be of such size as can be easily handled, enabling the masons to secure, beyond doubt, a good bedding, and each stone, with the exception of the smaller ones used for filling up and leveling up cavities, should be so selected or shaped as to present a good bed which will form a solid joint with the level surface prepared to receive it; all projections presenting too irregular outlines should also be hammered off. These conditions do not admit of large sizes for the stones used, but their limits of size cannot be absolutely determined in advance, and they may vary with the quality of the stone used.

Large stones, as generally prepared for rubble work, cannot be properly bedded, and better and more expeditious work will be secured by the use of smaller ones. With few exceptions, the largest stones should not exceed two tons and the largest proportion should be much smaller. The face on the water side should be made as water-tight as possible, either by a careful pointing of Portland cement or by the introduction of a special impervious facing.

Especial care will be necessary in designing the masonry of the Gate House, which must not present large masses of masonry different in character from that used in the body of the Dam, without forgetting that most of its parts will support heavier pressures.

The Gate House, however, is at a point on the side of the valley where the risks of unequal settlement are very much diminished by the reduced height of the structure.

The same precaution must be used on the other side, where an opening must be provided for the flow of the river during construction.

No special mention of the cut stone masonry to be used for the cornice and parapet work, for the gate house, for the overflow, &c., is necessary here.

This report would not be complete without a few words in connection with the part of the work which will precede the building of the masonry.

Existing dams furnish no precedents of the deep excavation necessary

here to reach the bed rock, and we find but few references to the manner of preparing the latter to receive the masonry.

How to make successfully that deep excavation and to take care of the river during the excavating operations, and during the first part of the construction, is the problem presented.

One plan originally suggested consisted in building across the valley, in heavily timbered trenches, and, from the rock bottom to the river bed, two walls of considerable thickness, forming the up-stream and down-stream faces of the dam respectively; under the protection of those walls, the earth could be removed between them and the wall completed.

Against that plan several important objections are found:

In view of the depth of the excavation, which is, at the lowest point of the valley, more than eighty feet deep, and in view of the inclination of the outer faces of the dam, these walls should be very thick, and the trenches containing them correspondingly wide. The excavation of such wide trenches, and their protection by timber, would be very costly, and would especially take a very long time.

The pressures at the bottom of the trench would be such as to require very heavy timbers placed very close together, thus forming an obstacle to the proper inspection, and to the expeditious and satisfactory laying of the masonry between them. Moreover, the rock bottom, under those difficulties, could not be properly examined and prepared. It is obvious, also, that the Dam would then be erected in its lower part in three parallel and disconnected vertical slices, instead of progressing regularly upward as it should be to secure the gradual settlement of the mass.

Notwithstanding the great depth of the excavation, and the evident difficulties accompanying such an operation, the most advisable manner to proceed is to open it without side protection, and to maintain it by means of long slopes on each side. The free access thus procured to every part of the work will allow of the use of powerful mechanical means for removing the earth, and the cost of the excess of excavation, due to the formation of slopes, will be more than compensated by the rapidity and simplicity of the operations, which will obviously lower the cost of the cubic yard excavated, and shorten very much the time necessary to complete the work.

Another advantage of as great value will result from the unobstructed condition of the bottom of the trench which will permit a perfect inspection and preparation of the bed rock and a thorough treatment of the seams and fissures which will, most probably, be found. The foot of the slopes, if necessary, can be locally removed, for a considerable distance, to uncover and fill up important

seams, which it would be found advisable to follow outside of the actual base of the dam. The numerous borings made in the valley show that the materials would be, on the whole, favorable for such an excavation, provided the trench is well drained.

In order to drain thoroughly the trench, it would be well (in addition to the means provided to take care of the river water) to sink in the bed of the river, a short distance from the upper edge of the excavation, several wells provided with pumping appliances, for the purpose of keeping the slopes dry.

It is well understood that the unusual depth of the proposed trench, and the constant presence above it of a body of water which, at times, may be very large, constitute difficulties of no mean magnitude, and that great care and foresight will be necessary to achieve success ; but this mode of treating the question appears to be the most rational under the circumstances.

Closely connected with this subject is that of the conveyance of the river water over the work while the excavating and building operations take place below the level of the river bed.

It is proposed to build two temporary wooden dams of little height, one above and one below the location of the dam, and to connect them by a water-tight timber flume, the distance of these dams from the edge of the trench to be determined later from a close examination of the grounds, to be not less in each case than, say, 200 feet.

The dams should be of such height as to correspond to the highest water mark of the flume.

What should be the flowing capacity of the flume ?

The records show an instance where the maximum flow of Croton River was about 8,000 million gallons in 24 hours.

The construction and maintenance of a wooden flume (not less than 200 feet wide), as formerly proposed, capable of safely containing such a stream, would be very expensive. It would require (supposing it to be practicable) great vigilance to keep it water-tight, owing to the constantly increasing depth of the excavations and to the consequently frequent change of supports. It would be, at best, a constant obstruction to the rest of the work, and should not be attempted.

In building operations of this character it is unavoidable that the builder should take some risks; but they must be reduced to a minimum within the limits of a reasonable outlay, and the writer thinks that the following manner of controlling the water, which is much less expensive, would afford more chance of protection from the water, although, in some contingencies, it may involve the temporary flooding of the work.

Instead of maintaining the protecting flume over the work it is proposed to

place it at one side, wholly or partially on a shelf of rock prepared, where necessary, for the purpose. The borings show that the north side would admit of such a treatment.

As to its flowing capacity, it would be sufficient to make it equal to the volume of the average spring freshet of the Croton River, corresponding to a width of about 60 feet; but the flume must be so designed that if the river rises beyond its capacity, ample channels, prepared for that purpose on the rocky hillside, must be opened to safely fill up the trench, and allow the freshet to pass over the work without causing more than repairable damage.

Time and money would then be required to restore the excavation to working order, but this risk is preferable to the constant maintenance of the larger channel during several years.

As the work of masonry comes up above the level of the river bed, an arched opening can be left for the passage of the flume, to be filled afterwards when the dam is nearly completed.

After the earth is removed from the proposed foundation on the bottom and sides, all loose or doubtful ledge must be removed until sound rock is found, or until such depth is reached as will be judged sufficient for securing a water-tight connection between the dam and the rock. For the purpose of insuring imperviousness, and of increasing the resistance of the structure to sliding, the surface of the rock must present numerous projections which need not be regular. In case the rock comes out with smooth surfaces, it would be well to excavate one or more wide longitudinal trenches. It is obvious that any seams that may be encountered must be carefully followed and obliterated by such means as may be suggested by local conditions. If seams and fissures extend to such distance as may require a more thorough treatment, it may be necessary, at places, if not on the whole extent of the work, to penetrate the rock deeply on the bottom and sides with a central trench to be filled afterwards with masonry.

It need not be said that the cements to be used in the construction must be well selected. It would be well, in order to secure perfect work at the connection of the structure with the rock, to use imported Portland cement at all places wherever such connection is to take place, and in their vicinity, the mortar being in the proportion of one of cement to two of sand.

The same mortar would also be preferable for the work to be done in the last season on the upper part of the dam and elsewhere, on account of the well-known property of the Portland cement to acquire hardness in shorter time than the domestic brands, thus securing greater power of resistance for the masonry, if it happens to be subjected to full pressures immediately after the dam is put into service.

Portland cement mortar should also be used for the pointing of the outer joints.

For the bulk of the dam, a mortar made of the same proportions of good domestic cement or a mortar of Portland cement in the proportion of three to one would be sufficient.

All the statements advanced, the conclusions reached, and the recommendations made in this report are presented on the supposition that every part of the work is to be executed with care, under intelligent and able superintendence; on that condition will depend the suceess of the undertaking.

The extensive borings made preparatory to the designing of the plan, especially the two diagonal borings made parallel with the sides of the valley under its whole width, seem to indicate the presence of good compact rock; the surface indications are also favorable, and although some difficulties are anticipated in preparing the foundations, it is confidently expected that they can be overcome.

Under the pressures to which the dam will be subjected, some leakage may be expected; such has been the case for several of the high masonry dams now in existence. If the leakage takes place through the masonry, experience shows that it diminishes gradually, and in regard to the grounds adjoining the dam, if the earth which covers them is not sufficient to prevent water from percolating through fissures that may exist in the rock, it is reasonable to expect that those percolations can be located and stopped.

In order to do full justice to this important subject, and to the investigation made of it, it would have been necessary to enter into more details in regard to the numerous questions which have been examined; but a complete record of them would have entailed many technical references hardly compatible with the scope and intent of this report,

Before concluding, I desire to record my indebtedness to Mr. Edward Wegmann, Jr., Division Engineer, whose work in connection with the mathematical part of this research I have already mentioned, and to Mr. Ira A. Shaler, Assistant Engineer, for their valuable assistance.

Respectfully submitted,

A. FTELEY,

Consulting Engineer.

1 Name of Dam.	2 Location.		18 Weight of masonry per cubic foot.	19 Area of water-shed in sq. miles.	20 Area of Reservoir in acres.	21 Capacity of Reservoir in million gallons.	22 Use of crest.	23 Length of waste weirs. Ft.	24 Max. height of water on weir. Ft.
Furens	St. Etienne, France	Water Pow	d, 125 lbs...	9.1	....	340	Roadway...	....	....
Villar	Lozoya, Spain	Water	143 lbs....	....	....	4,400	" ...	197	8.3
Gileppe	Verviers, Belgium	Water Pow	d, 143.5 lbs.	15 4	200	3,230	" ...	(2) 82 each..	6.5
Ban	France		....	....	....	....	" ...	....	...
Vyrnwy	Vyrnwy River, Wales	Water	160.8 lbs...	....	1,115	....	Overflow ...	....	....
Zola	Near Aix, Provence, France	Water er a	d, 137.1 lbs.	15.4	....	660	....	....	....
Habra	Province of Oran, Algiers	Irriga	d, 152 lbs...	....	7,950	....	Foot-path ..	410	5.2
Komatau	Assibach, Bohemia		....	....	....	358	Roadway...	....	....
Ternay	Annonay, France	Powe	d, 167 lbs...	....	675	....	....	197	....
Hamiz	Algeria	Irriga	....	54	....	3,430	....	....	....
Grancheurfas	"		....	....	....	4,224	....	....	....
Tlelat (to be raised higher)	"	Wate Irri	....	50	....	145	....	....	....
Djidionia (to be raised higher)	"		....	327	...	528	....	....	....
Boyd's Corners	Putnam Co., N. Y.	Wate	....	....	279	2,723	....	100	....
Geelong	Victoria, Australia	Wate	143 lbs....	4.7	26	142	Overflow ...	30	....
Puentes	Province of Sorca, Spain	Irriga	....	....	....	....	....	....	....
Alicante	Province of Alicante, Spain	Irriga	....	....	....	980	....	....	....
Val del Infierno	Province of Sorca, Spain	Irriga	....	....	...	....	....	....	....
Lozoya	Madrid, Spain	Wate	....	....	....	....	....	27.5	11
Nijar	Province of Almeria, Spain	Irriga	....	....	....	3,960	....	....	5.3
Elche	Province of Alicante, Spain	Irriga	....	....	....	...	....	....	....
Almanza	Province of Valencia, Spain	Irriga	....	....	....	....	....	39	6.6
Near San Mateo. recently begun	California, U. S. A.	Water Fra	....	24 (takes surplus water from large additional area).	1,800	32,000	....	....	....

TABLE II.

SYNOPSIS OF DATA—THEORETICAL PROFILE—QUAKER BRIDGE DAM.

1	2	3	4	5	6	7	8	9	10	11	12	13
Elevation of Joint (Croton Datum).	Depth of Water at Joint.	Offset on up-stream face from previous Joint.	Distance from up-stream end of joint to point at which a vertical through the centre of gravity intersects the joint. (u′)	Distance from down-stream end of joint to point at which the resultant of weight and water pressure intersects the joint. (u)	Distance between the points designated in columns 4 and 5. (c)	Total length of joint equals sum of columns 4, 5 and 6. (l)	Total area in sq. ft. equals total weight in cu. ft. above joint. (W)	Horizontal thrust of water above joint.	Pressure at up-stream end of joint. (p′)	Pressure at down-stream end of joint. (p)	Ratio of horizontal thrust of water to weight of masonry equals column 9 divided by column 8.	Factor of safety against overturning.
Ft.	Ft.	Ft.	Ft.	Ft.	Ft.	Ft.	Sq. ft. Cu. ft. Masonry.	Cu. ft. Masonry	lbs. per sq. ft.	lbs. per sq. ft.		
171.3	34.7	0	10.0	6.7	3.3	20	834	241	6,516	13,031	.29	3.00
156	50	0	10.5	8.7	7.0	26.2	1,187	500	11,328	14,156	.42	2.24
136	70	0	12.4	12.5	12.5	37.4	1,823	980	15,234	15,234	.54	2.00
116	90	2.3	17.8	17.8	17.8	53.4	2,731	1,620	15,984	15,984	.59	2.00
96	110	2.0	23.7	25.2	22.3	71.2	3.977	2,420	17,453	16,391	.61	2.13
76	130	3.4	31.7	35.1	26.1	92.9	5,618	3,380	18,462	16,384	.60	2.34
56	150	3.4	40.1	45.3	29.2	114.6	7,698	4,500	19,930	17,078	.57	2.55
35	171	3.6	49.1	56.1	32.2	137.4	10,339	5,848	21,822	18,219	.56	2.74
15	20 191	3.4	58.8	65.9	34.4	159.1	13,304 13,508	7,216	23,641	20,084	.53	2.92
−5	40 211	3.4	69.9	75.4	35.5	180.8	16,703 17,519	8,584	25,466	22,680	.49	3.13
−25	60 231	3.4	81.6	84.8	36.1	202.5	20,536 22,372	9,952	27,313	25,691	.44	3.35
−52	87 258	4.6	97.7	97.7	36.4	231.8	26,399 30,260	11,799	30,000	30,000	.39	3.68

NOTE.—Small figures in column 2 show depth of water on down-stream side of dam.

Small figures in column 8 represent weight of masonry (equals area masonry) *exclusive of gravel on slopes.*

Large figures in Column 8 below elev. 35 represent total weight of Masonry plus gravel; but in *area* they represent the area of the masonry plus 0.94 of the area of the gravel.

T COMMISSION

RAMS

ETICAL TYPES

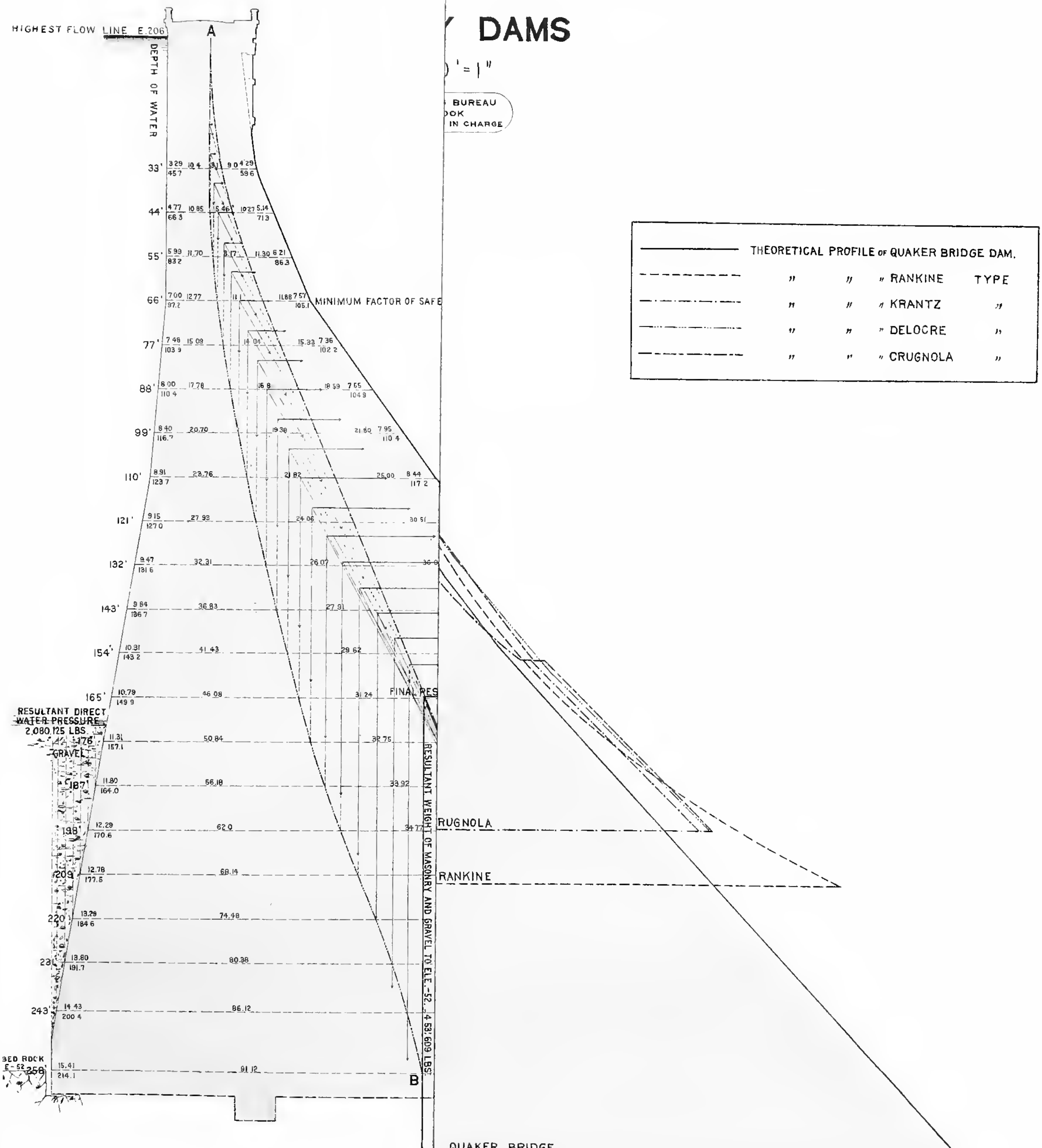
Y DAMS

) ' = 1 "

BUREAU
OOK
IN CHARGE

SHEET №3

TO ACCOMPANY REPORT OF CONSULTING ENG^R



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